Seismic Pier Design
for
Steel Pipe Pile Extensions with Concrete Cap Beam

State of Alaska

Department of Transportation &
Public Facilities

Bridge Section

Overview

- The purpose of this document is to assist in the design and detailing of Multiple Column / Pile Extension Piers

- The basis of this document is founded on the “Full-Scale Test of a Three Column / Pier Cap Bridge Substructure System Under Simulated Seismic Loading” by Seible, et al.

Typical Pier

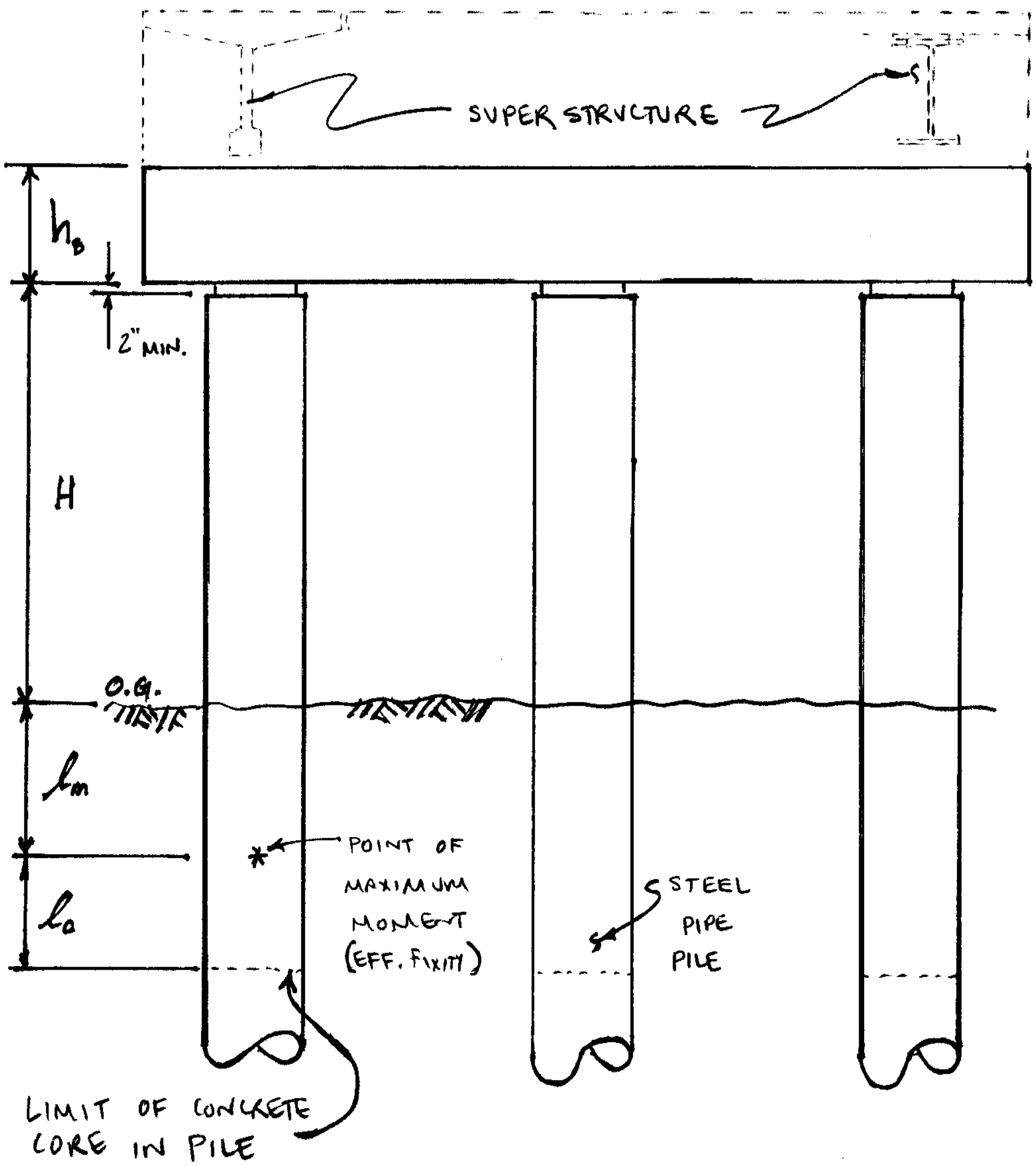


Figure 1

Step 1

- Collect the dead load forces in the pier cap and columns due to structure self weight, asphalt, utilities, etc

- These forces should include:

  P - axial
  V_y and V_z - shear
  M_y and M_z - moment

Step 2

- Collect the seismic forces in the pier cap and columns from multimodal computer analysis or other methods

- These forces should include:

  P - axial
  V_y and V_z - shear
  M_y and M_z - moment

- Consider both Load Combination I (100%L + 30%T) and Load Combination II (100%T + 30%L)

Step 3

- Determine the combined axial, shear and moment forces

- Note that the response modification factor, R, applies only to seismic moments in ductile members (i.e. where plastic hinges form)

$$M_{design} = \sqrt{\left[\frac{Meq\text{-}z}{R} + Mdl\text{-}z\right]^2 + \left[\frac{Meq\text{-}y}{R} + Mdl\text{-}y\right]^2}$$

$$V_{design} = \sqrt{\left[Veq\text{-}z + Vdl\text{-}z\right]^2 + \left[Veq\text{-}y + Vdl\text{-}y\right]^2} < Vp$$

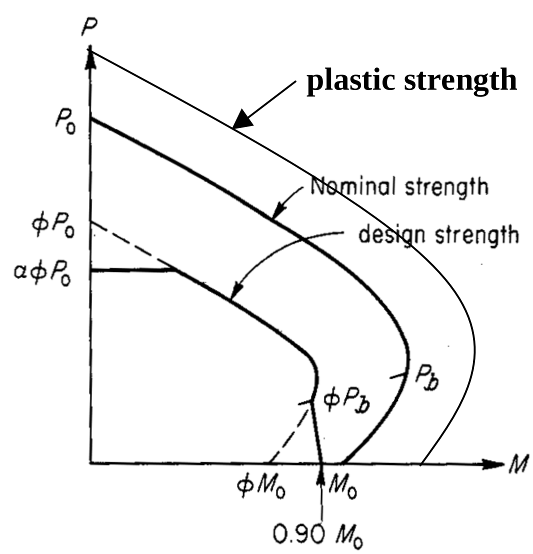
$$P_{design} = Pdl \pm Peq$$

Step 4

- Using the worst case load combination, determine the amount of longitudinal reinforcement required in the column, A_{sc}

- Do not over reinforce the column - this will lead to more cap beam and joint reinforcement

- Use ϕ factors as defined in AASHTO



Step 4

- There are many computer programs to aid in the design of concrete columns,

Recol_M	Imbsen & Associates
ULTCOL	Washington DOT

- Print out the P-M interaction information for later use in the cap beam design

- Note that AASHTO specifies a column reinforcement ratio

$$1\% < \rho < 4\%$$

but 3% is a practical upper limit due to joint reinforcement limitations

Step 5

- The ultimate applied shear, V_{ult}, is the <u>minimum</u> of either the design EQ shear or the shear associated with plastic hinging of the column, V_p

- Include the column overstrength factor for the concrete “gap” portion of 1.3 and 1.25 for the steel pipe when calculating V_p

- If the required moment capacity of the column is close to the balance moment, use the balance moment in subsequent calculations

Step 5

- The shear associated with plastic hinging is calculated as shown below. *It is good practice to use the V_p for design if practical*

$$V_p = \frac{M_1 + M_2}{H_e}$$

where:

M_1 = moment at top of column

= M_n * 1.3 - concrete column

M_2 = moment at bottom of column

= 1.25 * M_p - steel pipe

H_e = effective height of column

= H + l_m (see figure 1)

Step 6

- Determine the size and pitch of the spiral in the column

$$V_{ult} < \phi * V_n$$

where:

ϕ = 0.85 (16th) 0.9 (LRFD)

V_n = nominal shear capacity
see AASHTO code or
UCSD shear design
equations

D = column / pile diameter

Step 7

- Determine the amount of the cap beam steel required noting that:
- the height of the cap beam, h_b, must be greater than the development length of the column longitudinal steel and

$$D < h_b < D * 1.15$$

- the width of the cap beam, b_j, must satisfy the following:

$$D + 12 \text{ in} < b_j < D + D/2$$

- Use the maximum overstrength moment of the column to “load” the cap beam (i.e., M_p at P_{max})

Step 7

- The required development length of the longitudinal column reinforcement is:

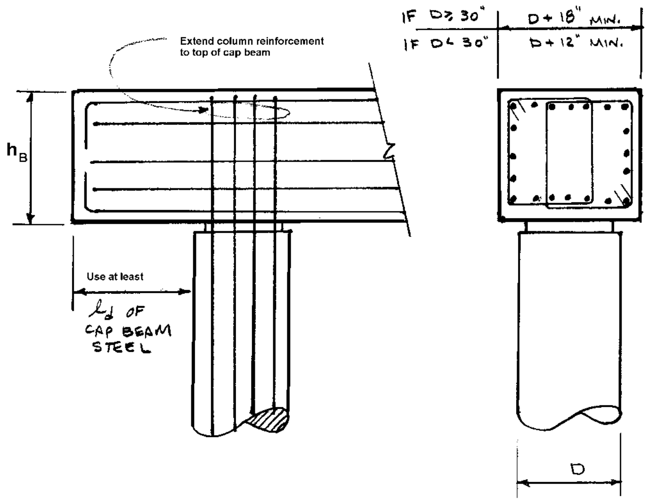
  $l_d = 0.025 * d_b * F_y / \sqrt{(f'_c)}$

  where:
  d_b = diameter of bar
  F_y = rebar yield strength (psi)
  f'_c = concrete strength (psi)

- To use this length, welded hoop or spiral reinforcement must be used in the joint (defined in step 9)

- Always extend longitudinal column bars to the top of the cap beam

Step 7



$$\varphi * M_n > M_{ult}$$

Where: $\varphi = 0.9$ for bending

M_n = nominal bending capacity

$M_{ult} = M_p + M_{dl}$

M_p = plastic moment capacity of

column associated with P_{max}

Check that the cap beam is not over or under reinforced and that temperature and shrinkage steel requirements are satisfied

Step 8

- Determine the size and spacing of shear stirrups required in the cap beam

$$V_{ult} < \phi * V_n$$

where:

ϕ = 0.85 (16th ed.) 0.9 (LRFD)

$V_n = V_c + V_s$

$V_{ult} = V_{dl} + V_{p\text{-}cap}$

$V_{p\text{-}cap}$ = shear in cap beam due to plastic hinging of column

$= \dfrac{1.5 * M_p}{S_{col}}$ (approx.)

- Use shear at "d" from face of column

Step 9

- Determine the size and spacing of welded hoops required in the joint region of the cap beam

- This steel is needed to provide development length and confinement for the column longitudinal steel

$$s = \frac{4*A_h}{D'*\rho_s} < \frac{h_b}{4}$$

where:

s = welded hoop spacing

A_h = area of welded hoop

e.g. #5 hoop = 0.31 in^2

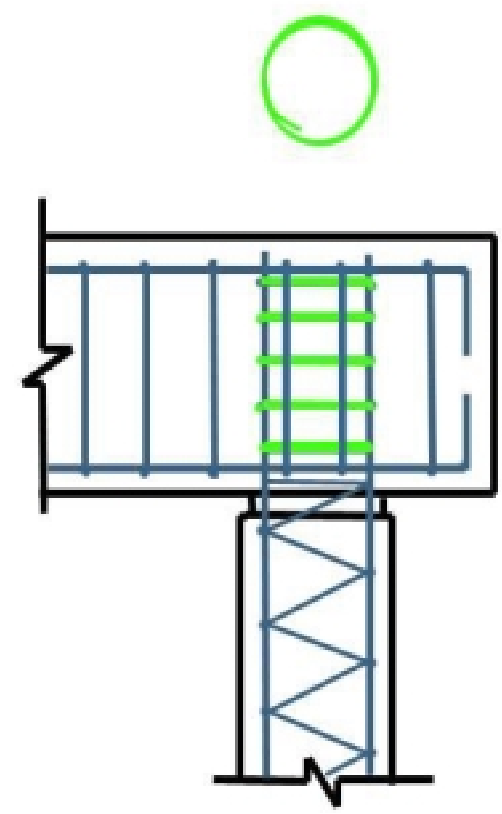
Step 9

- Continued

  l_a = anchored length of A_{sc}
  A_{sc}= Area of column
  longitudinal steel rebar
  h_b = height of cap beam
  λ_o = overstrength factor
  = 1.4
  $\rho_s = 0.3*\lambda_o*A_{sc} / l_a^2 > 3.5*\sqrt{(f'_c)}/ Fy$
  D' = core diameter of column

- Provide a cap beam height greater than the anchorage length required for the column longitudinal steel (see step7)

Welded Hoop Steel

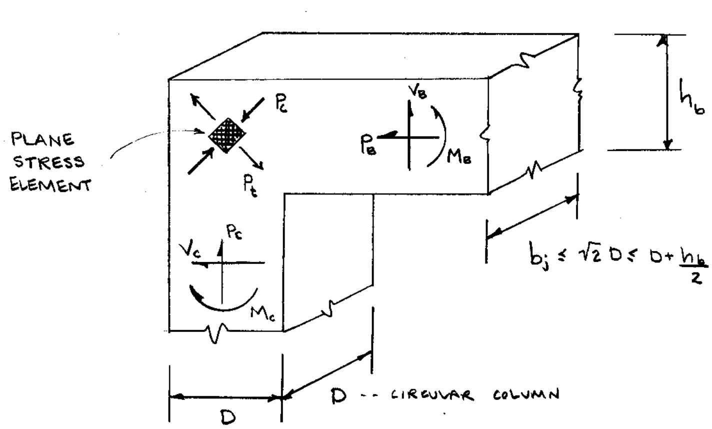


$$S < \frac{4 * A_h}{D' * \rho_s} < \frac{h_b}{4}$$

Typically these bars will be field welded after placement

Step 10

- Determine the average principal tensile stress in the joint



$$p_{c,t} = \frac{(f_v + f_h)}{2} \pm \sqrt{\frac{(f_v - f_h)^2}{4} + v_j^2}$$

where:

$$f_v = \frac{P_c}{b_j * (D + h_b)}$$

Step 10

- Continued

$$f_h = \frac{V_c}{b_j * h_b}$$

$$v_j = \frac{M_c}{h_b * D * b_j}$$

M_c = moment in the column
V_c = shear in the column
P_c = axial load in the column
D = column diameter
h_b = height of cap beam
b_j = width of cap beam and
$< \sqrt{2} * D$
$< D + h_b$

- Always check your signs (+/-)

Step 10

- Use M_c,V_c, and P_c which result in maximum principal tension, p_t
- If the principal tension (p_t) is greater than $3.5*\sqrt{(f'_c)}$ then additional joint reinforcement is required - that is:

  If $p_t < 3.5* \sqrt{(f'_c)}$ then done

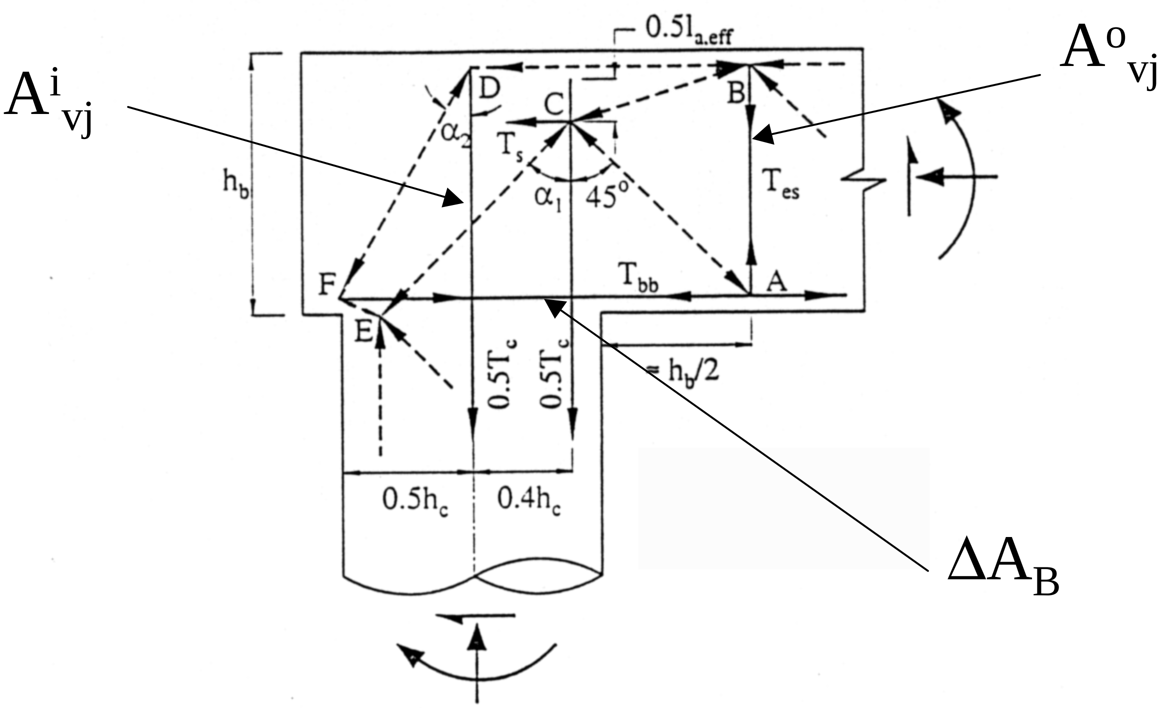
  If $p_t > 3.5* \sqrt{(f'_c)}$ then provide the additional reinforcement defined in the following steps

  If $p_t > 15* \sqrt{(f'_c)}$ then joint will not work - try different pier geometry

Strut and Tie Model

- The model developed by UCSD (shown below) was used to generate the following joint design procedure

- Area of steel to resist tensile forces (T_{es}, T_{bb} and T_c) is determined from joint geometry and reinforcement pattern



Step 11

- Determine the extra amount of shear reinforcement (paired hoops) required outside the joint region, A^o_{vj}

- Space the stirrups evenly in a region equal to the cap beam height. Total area A^o_{vj} to each side of the column

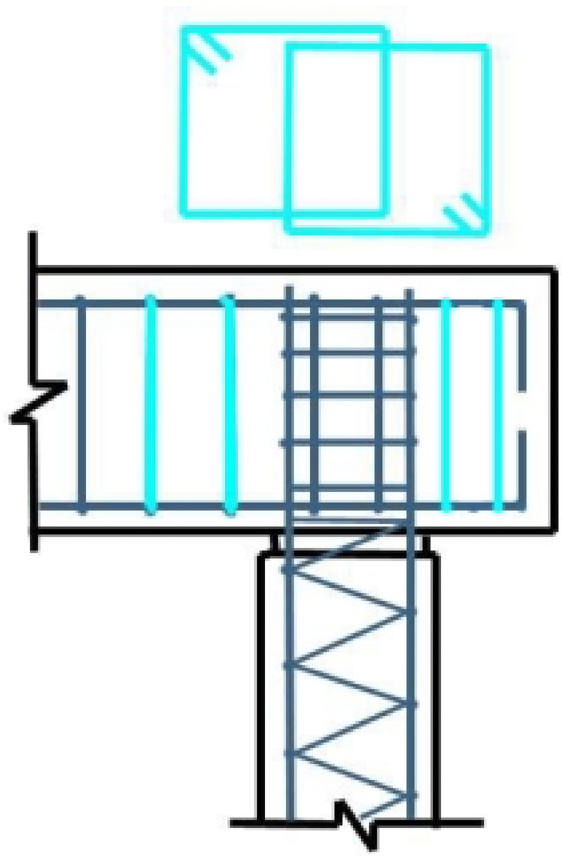
$$A^o_{vj} > 0.125 * \lambda_o * A_{sc}$$

where:

A_{sc} = area of column longitudinal steel

λ_o = overstrength factor = 1.4

Shear Reinforcement Outside Joint



$$A_{vj}^{o} > 0.125 * A_{sc} * \lambda_o$$

Put the paired hoops on each side of the joints

Step 12

- Determine the amount of shear reinforcement (paired hoops) required in the joint, A^i_{vj}
- Space these bars evenly within the joint region over the column

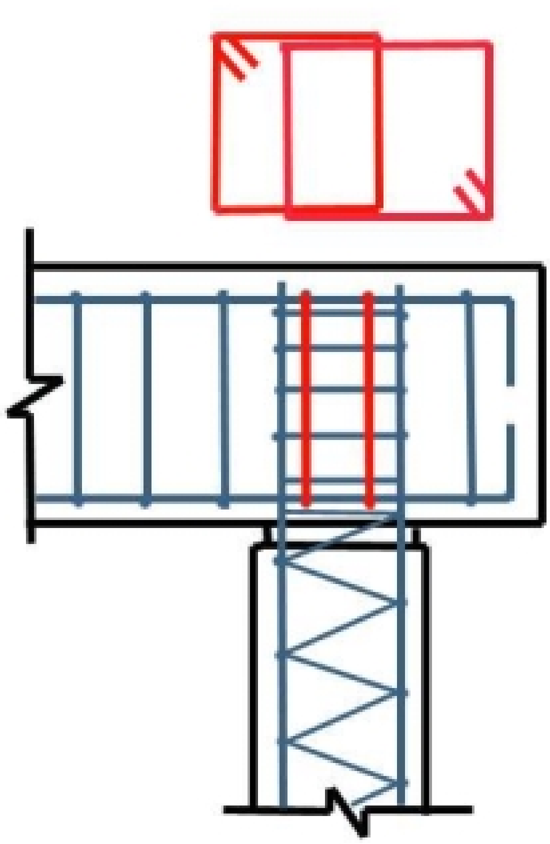
$$A^i_{vj} > 0.095 * \lambda_o * A_{sc}$$

where:

A_{sc} = area of column longitudinal steel

λ_o = 1.4

Shear Reinforcement Inside Joint



$$A_{vj}^{i} > 0.095 * A_{sc} * \lambda_o$$

Space paired hoops evenly within joint region

Step 13

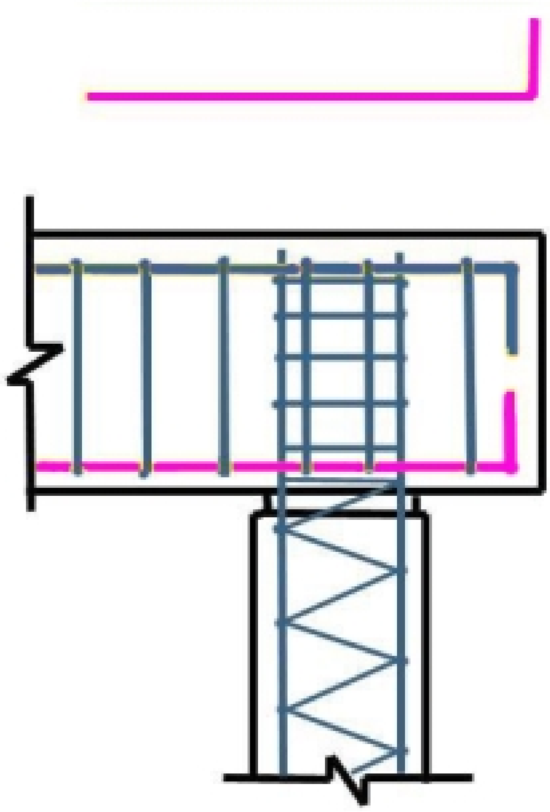
- Additional top and bottom longitudinal reinforcement is required to develop the joint strut-and-tie mechanism

- Add the following amount of cap beam longitudinal steel, ΔA_b, *in addition to what is required to resist bending alone*, to both top and bottom

$$\Delta A_b > 0.17 * \lambda_o * A_{sc}$$

where:
A_{sc} = area of column longitudinal steel
$\lambda_o = 1.4$

Additional Longitudinal Beam Reinforcement



$$\Delta A_b > 0.170 * A_{sc} * \lambda_o$$

Put additional longitudinal bars on top AND bottom

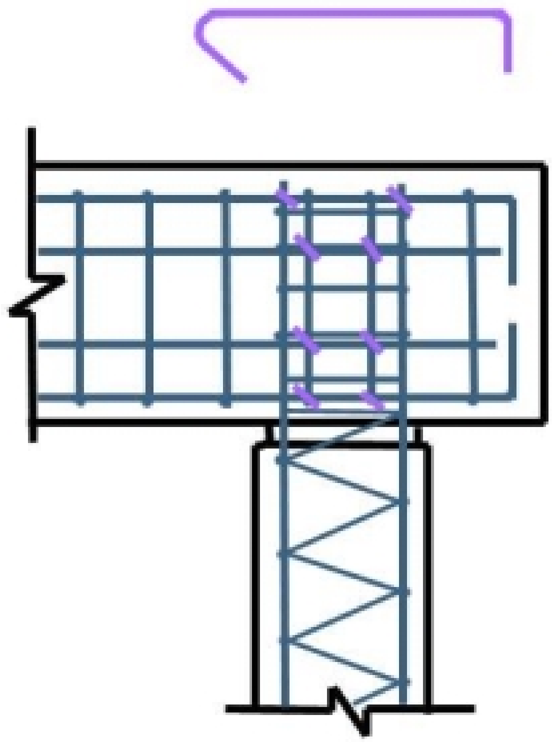
Step 14

- Provide seismic “J’ bars within the joint regions to prevent buckling of the longitudinal steel in the cap beam and to provide additional confinement of the joint region

- Two or three “J” bars per longitudinal cap beam bar *should* be adequate for most cases

- Space the bars evenly within the joint so as to prevent buckling of longitudinal cap beam bars

Transverse Seismic “J” Bars



Place two or three three “J” bars per longitudinal cap beam bar within joint region

Could use welded, headed bars if desired

Detailing Notes

- Provide concrete core down pipe pile below the depth of effective fixity (point of maximum moment) by at least 3 pile diameters or to the point where the pile moment is about half the maximum moment
- Make sure that the longitudinal cap beam bars are fully developed - may need to provide 90^{o} hooks
- Use headed reinforcement in place of the “J” bars and on the ends of the longitudinal cap beam bars if space is tight

Detailing Notes

- Use paired shear stirrups (hoops) in pier cap beams. This provides better confinement of concrete and a more even distribution of steel within joint region to better carry the loads
- Generally, more smaller bars are better than few larger bars for serviceability. However, you must still meet bar spacing requirements for concrete placement
- Although the earthquake load case often governs the pier design, you must still examine the other load combinations (strength and serviceability)

References

- **Nilsson et al** Reinforced Concrete Corners and Joints Subjected to Bending Moment (1976)
- **Park and Paulay** Reinforced Concrete Structures (1976)
- **Schlaich, Schäfer, and Jennewein** Towards a Consistent Design Methodology (1987)
- **Collins and Mitchell** Prestressed Concrete Structures (1991)
- **James G. MacGregor** Reinforced Concrete; Mechanics and Design (1992)
- **Priestley, Seible, and Calvi** Seismic Design and Retrofit of Bridges (1996)
- **AASHTO** AASHTO LRFD Bridge Design Specifications (1996)
- **ASCE-ACI Committee 445** Recent Approaches to Shear Design of Structural Concrete (1998)
- **Silva, Sritharan, Seible, and Priestley** Full-Scale Test of the Alaska Cast-in-Place Steel Shell Three Column Bridge Bent (1999)